Lifetime Reliability Assessment of Concrete Slab Bridges

Thoft-Christensen, Palle

Publication date:
1997

Document Version
Publisher's PDF, also known as Version of record

Link to publication from Aalborg University

Citation for published version (APA):
Thoft-Christensen, P. (1997). *Lifetime Reliability Assessment of Concrete Slab Bridges*. Dept. of Building Technology and Structural Engineering. Structural Reliability Theory Vol. R9717 No. 168

INSTITUTTET FOR BYGNINGSTEKNIK
DEPT. OF BUILDING TECHNOLOGY AND STRUCTURAL ENGINEERING
AALBORG UNIVERSITET • AAU • AALBORG • DANMARK

STRUCTURAL RELIABILITY THEORY
PAPER NO. 168

Presented at the NSF-Workshop on "Optimal Performance of Civil Infrastructure Systems", Portland, Oregon, USA, April 12, 1997

P. Thoft-Christensen
Lifetime Reliability Assessment of Concrete Slab Bridges
June 1997 ISSN 1395-7953 R9717

The STRUCTURAL RELIABILITY THEORY papers are issued for early dissemination of research results from the Structural Reliability Group at the Department of Building Technology and Structural Engineering, University of Aalborg. These papers are generally submitted to scientific meetings, conferences or journals and should therefore not be widely distributed. Whenever possible reference should be given to the final publications (proceedings, journals, etc.) and not to the Structural Reliability Theory papers.

Printed at Aalborg University

INSTITUTTET FOR BYGNINGSTEKNIK
DEPT. OF BUILDING TECHNOLOGY AND STRUCTURAL ENGINEERING
AALBORG UNIVERSITET • AAU • AALBORG • DANMARK

STRUCTURAL RELIABILITY THEORY
PAPER NO. 168

Presented at the NSF-Workshop on "Optimal Performance of Civil Infrastructure Systems", Portland, Oregon, USA, April 12, 1997

P. Thoft-Christensen
Lifetime Reliability Assessment of Concrete Slab Bridges
June 1997 ISSN 1395-7953 R9717

Lifetime Reliability Assessment of Concrete Slab Bridges

Palle Thoft-Christensen[1]

Abstract

A procedure for lifetime assessment of the reliability of short concrete slab bridges is presented in the paper. Corrosion of the reinforcement is the deterioration mechanism used is estimating the reliability profiles for such bridges. The importance of using sensitivity measures is stressed. Finally the procedure is illustrated on 6 existing UK bridges.

Introduction

This paper contains details of a methodology which can be used to generate Whole Life (WL) reliability profiles. These WL reliability profiles may be used to establish revised rules for Concrete Bridges. The paper is based on research performed for the Highways Agency, London, UK under the project DPU/9/44 *"Revision of Bridge Assessment Rules Based on Whole Life Performance: Concrete Bridges"*, (Thoft-Christensen & Jensen, 1996.4) and to some extent on (Thoft-Christensen et al. 1996.1) , (Thoft-Christensen, 1996.2), and (Thoft-Christensen et al. 1996.3).

Limit States

The lifetime reliability estimation is based on two ultimate limit states (ULS): collapse limit state (using yield line analysis) and a shear failure limit state. The reliability index $\beta_y(t)$ *for reinforcement yielding failure* is estimated using the yield line analysis program COBRAS produced at Cambridge University (Middleton, 1994). COBRAS contains a large failure mode library, and a sub-set of 8 failure modes relevant to bridge slab analysis has been selected, see figure 1. The set of failure mechanisms included may be extended for analysis of other types of bridges.

[1] Professor, Aalborg University, Sohngaardsholmsvej 57,
DK-9000 Aalborg, Denmark

modes relevant to bridge slab analysis has been selected, see figure 1. The set of failure mechanisms included may be extended for analysis of other types of bridges.

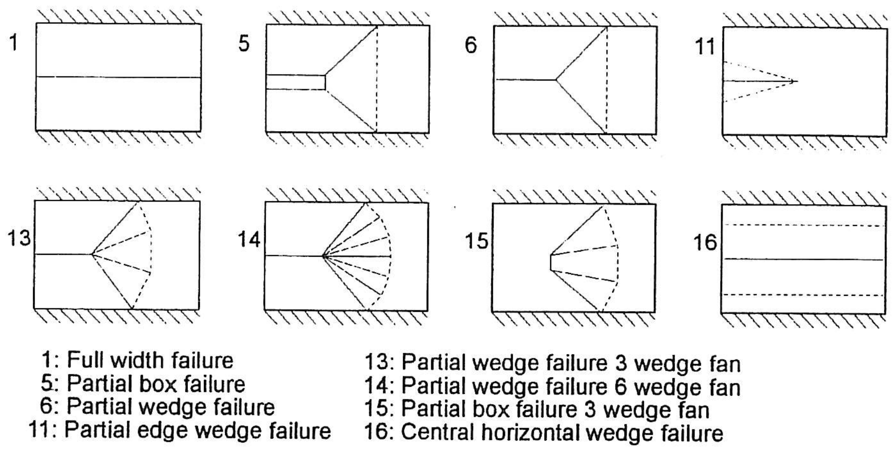

Figure 1. Relevant failure modes for bridge slab analysis. A full line indicates a sagging yield line and a dotted line indicates a hogging yield line.

With regard to beam/slab type bridges the COBRAS program uses a simplified and conservative approximation technique for calculation of the flexural moment capacity of bridge decks with different cross-sectional depth such as beam/slab decks or decks with edge beams. The approximation used underestimates the actual strength of a section and is therefore a "safe" assumption.

The critical failure modes used in the estimation of the reliability profiles are the critical failure modes identified at time the $t = 0$. After corrosion is initiated the critical failure may change due to the reduced bending strength. However, in this study homogeneous corrosion is assumed, so it is unlikely that the critical failure modes (yield patterns) will change significantly. The reliability index $\beta_{sh}(t)$ for shear failure is based on a linear elastic analysis using a linear elastic finite element program. The probability of failure for a slab/beam bridge is estimated as a series system consisting of the two failure modes (bending and shear failure). However, the correlations between these failure modes is usually very close to 1 so the series system reliability index β^{sys} may be approximated by the minimum of the element reliability indices.

Reliability Analysis

The reliability analysis (element and system) is done using the programs (RELIAB01, 1994) and (RELIAB02, 1994). The RELIAB and COBRAS programs have been interfaced and include an optimisation algorithm to determine the optimal yield line pattern for each iteration of the reliability analysis, see also (Thoft-Christensen , 1989). The estimation of the deterioration of the steel reinforcement is based on the program (CORROSION, 1995).

The safety margin used for yielding (yield line) failure is

$$M_1: \; g_1(\cdot) = Z_1 E_D - W_D \tag{1}$$

where E_D is the energy dissipated in the yield lines (or plastic zones) and W_D is the work done by the applied load. Z_1 is a model uncertainty variable related to the calculation of the energy dissipation.

The safety margin used for shear failure is

$$M_2: \; g_2(\cdot) = Z_2 V_{j,ult} - V_j \tag{2}$$

where $V_{j,ult}$ is the ultimate shear strength at section j, V_j is the shear force at section j and Z_2 is a model uncertainty variable related to ultimate shear strength.

The basic variables used in the yield line ULS are: thickness of slab, cube strength of concrete, density of concrete, depth of reinforcement, yield strength of reinforcement, and two load parameters. The stochastic variables used in the shear limit state are: thickness of slab, cover on reinforcement, concrete cube strength, yield stress of reinforcement, initial area of the reinforcement, density of concrete, static load factor, dynamic load factor, model uncertainty variable, and variables related to the chloride induced corrosion.

Modeling of Deterioration

In this paper only one deterioration mechanism is considered namely chloride induced corrosion of the reinforcement. When concrete is exposed to chloride it has become normal practice to describe the response of the concrete to the chloride exposure by its chloride profile, i.e. the distribution of the chloride content of the concrete in its near-to-surface layer or by the concentration-distance curve.

Estimation of the chloride profile is a very uncertain matter since it is controlled by a number of factors which are difficult to model. The origins of chlorides in concrete are usually divided into:

- chlorides incorporated in the concrete when it was mixed, e.g. from salty aggregates, salty mixing water, and admixtures containing chloride
- chlorides penetrating into the concrete from the environment, e.g. from seawater, and de-icing salts.

The information given by the exposure time and the achieved chloride profile of the concrete may be simplified by a few parameters sufficient to determine the shape of the chloride profile from a mathematical point of view. The controlling parameters with regard to the corrosion initiation time are the initial chloride content C_i, the chloride content at the surface C_0, and the chloride diffusion coefficient D_c. After corrosion has been initiated then the controlling parameter is the rate of corrosion i_{corr}.

Corrosion of the reinforcement is supposed to take place when the chloride concentration at the site of the reinforcement reaches a critical level C_{cr}. The corrosion due to chloride ingress will usually be pitting corrosion, i.e. a very localised corrosion of the reinforcement. When corroded rebars become pitted their properties change. Pitting is particular vicious because it is a localised and intense form of corrosion, and failures of the bar in question often occur with extreme suddenness. *For structures where ductility is needed the initiation stage of corrosion can be taken as the service lifetime* (Thoft-Christensen,1997). Furthermore, the cost of rehabilitation of a structure during the initiation period is generally small compared with the rehabilitation during the corrosion propagation period.

For a reinforced concrete slab bridge pitting corrosion of a single rebar or a few rebars will not drastically change the ductility due to the "parallel" behaviour of the rebars. Therefore, in this paper it is considered acceptable to model the corrosion as a uniform corrosion of the rebars to avoid the difficult task of including pitting corrosion. In the modelling of the deterioration sufficient ductility is assumed preserved to justify application of the yield line theory.

The rate of chloride penetration into concrete is modelled by Fick's law of diffusion

$$\frac{\delta c(x,t)}{\delta t} = D_C \frac{\delta^2 c(x,t)}{\delta x^2} \tag{3}$$

where D_C is the chloride diffusion coefficient, x is the distance from the surface and t is the time. The solution of the equation (3) is

$$C(x,t) = C_0 \left\{ 1 - erf\left(\frac{x}{2\sqrt{D_C t}} \right) \right\} \tag{4}$$

where $C(x,t)$ is the chloride content at the distance x from the surface and at the time t. C_0 is the chloride content on the surface. The corrosion initiation period is

$$T_I = \frac{(d_I - D_I/2)^2}{4D_C} (erf^{-1}(\frac{C_{cr} - C_0}{C_i - C_0}))^{-2} \tag{5}$$

where C_i is the initial chloride concentration, C_{cr} is the critical chloride concentration, and $d_I - D_I/2$ is the concrete cover.

The diameter $D_I(t)$ of the reinforcement bars at the time t after initiation of corrosion can then be modelled by

$$D_I(t) = D_I - C_{Corr} i_{corr} t \tag{6}$$

where D_I is the initial diameter, C_{corr} is a corrosion coefficient, and i_{corr} is the rate of corrosion.

Three levels of deterioration are proposed based on a survey: low deterioration, medium deterioration and high deterioration.

Low:	Diffusion coefficient	D_C:	N(25.0, 5.0)[mm2/year]
	Chloride conc. , surface	C_0 :	N(0.575, 0.038) [%]
	Corrosion density	i_{corr} :	Uniform[2.0, 3.0] [μA/cm2]
Medium:	Diffusion coefficient	D_C:	N(30.0, 5.0) [mm2/year]
	Chloride conc., surface	C_0 :	N(0.650, 0.038) [%]
	Corrosion density	i_{corr} :	Uniform[3.0, 4.0] [μA/cm2]
High:	Diffusion coefficient	D_C:	N(35.0, 5.0) [mm2/year]
	Chloride conc. , surface	C_0 :	N(0.725, 0.038) [%]
	Corrosion density	i_{corr} :	Uniform[4.0, 5.0] [μA/cm2]

Realisations of these three models are shown in figure 2. The area of reinforcement is modelled deterministically using the equation

$$A(t) = \begin{cases} nD_i^2 \frac{\pi}{4} & \text{for } t \le t_a \\ n(D(t))^2 \frac{\pi}{4} & \text{for } t_a \le t \le t_a + D_i / \alpha \\ 0 & \text{for } t > t_a + D_i / \alpha \end{cases} \tag{7}$$
$$\text{where } D(t) = D_i - \alpha(t - t_a)$$

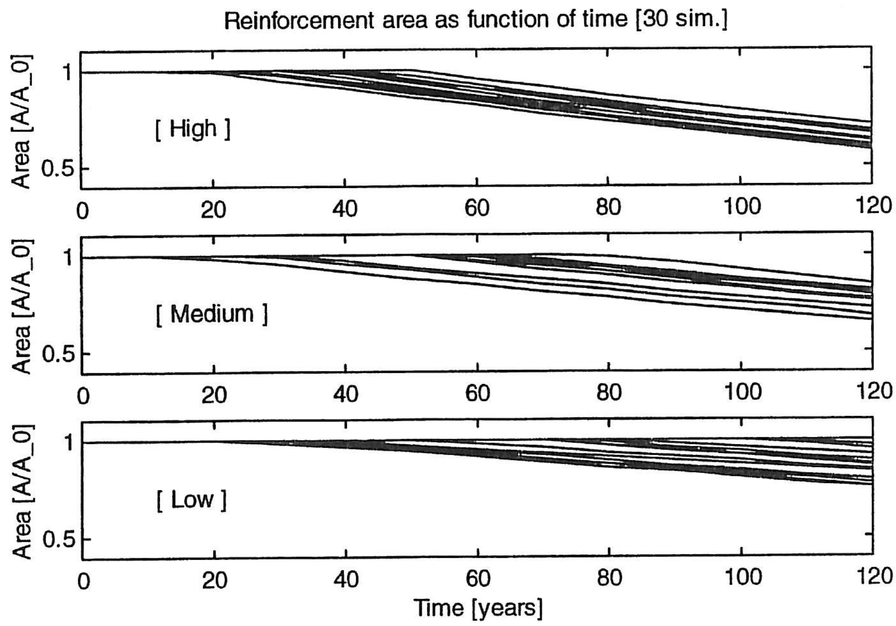

Figure 2. Normalised reinforcement area A / A_0 as a function of time for low, medium, and high deterioration..

Whole Life Reliability Profiles

The lifetime estimation of the reliability profiles is based on data from a number of UK bridges. Reliability profiles for the collapse limit state are calculated on the basis of a stochastic modeling of the following parameters: thickness of slab, cube strength of concrete, density of concrete, yield strength of longitudinal reinforcement, cover on longitudinal reinforcement, cover on longitudinal reinforcement, yield strength: transverse reinforcement, cover on transverse reinforcement, longitudinal reinforcement area, transverse reinforcement area, static load factor, static load factor, dynamic load factor, chloride concentration on surface, initial chloride concentration, diffusion coefficient, critical chloride concentration, corrosion parameter, and a model uncertainty variable.

Later in the paper the Whole Life Reliability profiles for 6 typical existing bridges in UK are shown. All bridges are "good" bridges in the sense that they are well maintained and no serious damage or deterioration has been observed during inspections.

Traffic Load

The general traffic highway load model in (Eurocode 1, Part 3, ENV 1991-3:1995) for lane and axle load is applied. The load effects produced by the Eurocode model (lane and axle load) are multiplied by a static load factor (extreme type 1) and a dynamic load factor (normal).

The load model operates with four load combinations:

- Lane 1 loaded alone (without adjacent traffic lane loading).
- All traffic lanes loaded, with loads in marked traffic lanes.
- All road width loaded, with traffic tightly packed side-by-side ("narrow lane width" case).
- Accidental load on footway

The F&N load model specifies statistical modification factors which are applied to the load effects (moments, shears, end reactions) produced by the load models. The statistical modification factors are used on the effects on components given by the EC load models.

The statistical modification factors are specified as statistical parameters (U, α) for Extreme type 1 distributions (Gumbel) for different load effects (moment, shear), different traffic volumes (motorway, main, minor roads), different spans (5,10,20,30,40,50 m) and for load combinations I, II and III.

A dynamic amplification factor is applied to the load effects produced by the load models. The amplification factor is given by a normal distribution for different road conditions (good, medium/poor) and lane 1 loaded alone and 1,2 or more lanes loaded

For the load combinations the most adverse location of loads is determined, i.e. the location of axle loads and the order of lane loading giving the most critical sections with regard to ULS.

The "design variables" can therefore be defined as: the location of axle loads (continuous), the order of lane loading (discrete) and the sections with critical ULS (discrete/continuous). Three options for determining an adverse location of load and the order of lane loading have been considered:

a) the location/order of loads is determined based on previous knowledge and experience, and the reliability assessment is made for this location/order of loads, or
b) the location/order of loads is determined using mean values of the loads by varying the possible "design variables" and the reliability assessment is performed using this fixed location/order of loads, or
c) the reliability assessment is performed by varying the possible "design variables", and the location/order of loads corresponding to the smallest reliability index is chosen.

Sensitivity Analysis

In this section three important sensitivity measures are discussed, namely the sensitivity factor, the reliability elasticity coefficient and the omission sensitivity factor. It contains a sensitivity analysis of one of the selected bridges. Similar sensitivity analyses have been performed for all bridges, but since the results are almost identical only the sensitivity analysis of one bridge is presented. The change of the sensitivity factors (alpha vectors) when deterioration starts is analysed and the elasticities with respect to mean values and standard deviations are commented on. Finally, the omission sensitivity factors are shown for this bridge.

Sensitivity analysis is a very useful tool in connection with data collection and bridge management. Sensitivity analysis can be used to decide whether a parameter can be modelled by a deterministic variable or whether a stochastic modelling is needed. It also gives information on the importance of the single parameters with regard to the reliability estimation, so that requirements for the data collection can be derived.

Let $\bar{\alpha} = (\alpha_1, .., \alpha_n)$ be the normal unit vector to the failure surface at the design point. The element α_i is then called the *sensitivity factor* of the stochastic variable i . α_i^2 is the fraction of the variance of the safety margin that originates from stochastic variable X_i if the stochastic variables are uncorrelated. If the stochastic variables are mutually dependent then α_i^2 is only an approximation of the variance.

The *reliability elasticity coefficient* associated with a parameter p (e.g. the mean value or the standard deviation of a stochastic variable or a constant in the failure function) is defined by

$$e_p = \frac{d\beta}{dp}\frac{p}{\beta} \tag{8}$$

It follows from this definition that if the parameter p is changed by 1 % then the reliability index β is changed by **Fejl! Bogmærke er ikke defineret** %.

The third sensitivity measure is called the *omission sensitivity factor*. It is introduced by (Madsen (1988). For a linear failure function and normally distributed stochastic variables the omission sensitivity factor ς_i is defined by

$$\varsigma_i = \frac{1-\alpha_i u_i^0}{\sqrt{1-\alpha_i^2}} \tag{9}$$

This factor gives the relative importance of the reliability index β assuming that the stochastic variable no. i is fixed on the value u_i^0, i.e. if it is considered as a deterministic parameter. If $u_i^e = 0$ i.e. stochastic variable no. i is fixed to its mean value then the reliability omission factor is

$$\varsigma_i = \frac{1}{\sqrt{1-\alpha_i^2}} \tag{10}$$

The sensitivity analyses show that for the failure mode bending it can be concluded that the yield strength and the depth of the longitudinal reinforcement, the static and the dynamic load factors and the model uncertainties are the important variables before the corrosion is initiated. After corrosion is initiated the critical chloride concentration and the corrosion parameter become significant stochastic variables.

Examples

The assessment procedure presented in this paper has been applied to 15 slab and slab/beam bridges in the UK. In this section the assessment of 6 of these bridges is shown.

Bridge 1:

Bridge 1 carries the A11 over a minor road. The bridge was constructed in 1975. The span is 12.80 m, the width 2x13.95 m with a skew of 21 degrees. The bridge consists of two identical parts where only one part is analysed. The bridge is designed for 45 HB load. The main dimensions and material data used in the analysis model are shown in Figure 3. The nominal cover on the longitudinal reinforcement is 34 mm.

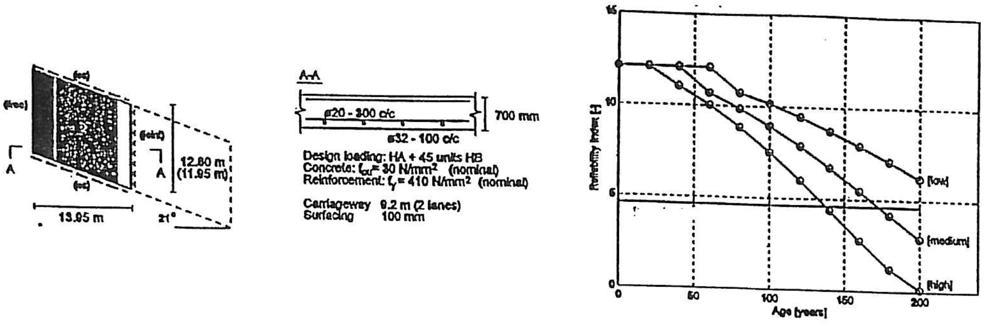

Figure 3. Bridge 1

Bridge 2

Bridge 2 carries the A45 over a river. The bridge was constructed in 1975. The span is 9.75 m and the width is 2 x 13.7 m. The bridge is designed for 45 HB load. The main dimensions and material data used in the analysis model are shown in Figure 4. The nominal cover on the longitudinal reinforcement is 40 mm.

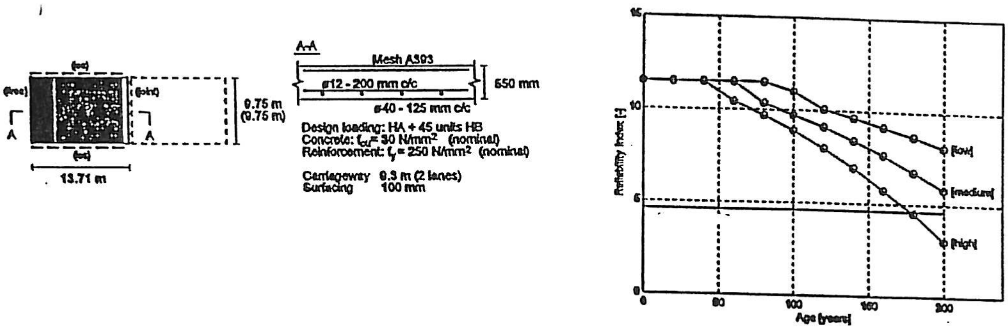

Figure 4. Bridge 2

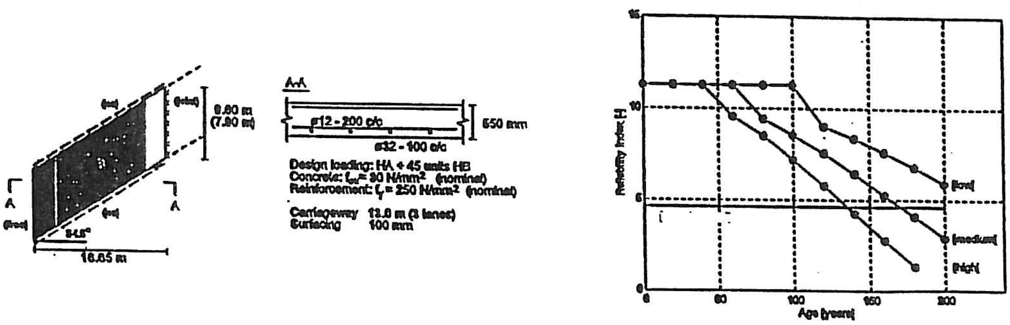

Figure 5. Bridge 3

Bridge 3

Bridge 3 carries the A45 trunk road over a river. The bridge was constructed in 1975, the span is 9.6 m, the width 2x18.65 m with a skew of 34.5 degrees. The bridge has a joint along the centreline, so only half the bridge is analysed. The main dimensions and material data used in the analysis model are shown in Figure 5. The nominal cover on the longitudinal reinforcement is 40 mm.

Bridge 4

Bridge 4 carries a principal road over the A34 trunk road. The bridge was constructed in 1973, the span is 20.2 m and the width 9.8 m. The bridge consists of two identical spans. Only one span is analysed. The bridge is designed to carry 37.5 HB load. The main dimensions and material data used in the analysis model are shown in Figure 6. The nominal cover on the longitudinal reinforcement is 40 mm.

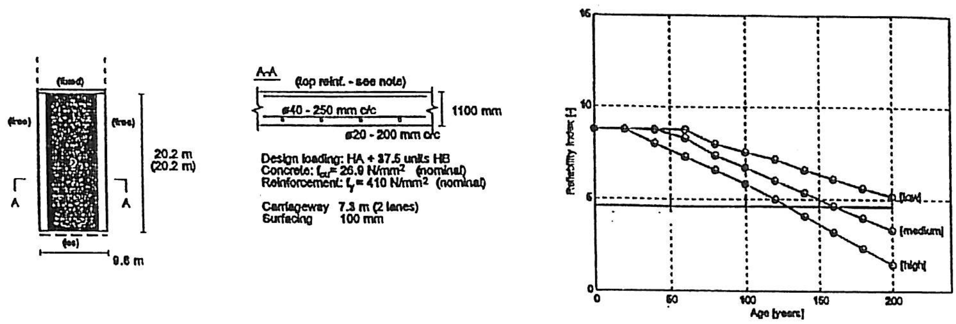

Figure 6. Bridge 4

Bridge 5

Bridge 5 carries the A12 trunk road over a river. The bridge was built in 1954, the span is 6.4 m and the width 18.2 m with a skew of 47 degrees. The bridge is designed to carry 45 units of HB load. The main dimensions and material data used in the analysis model are shown in Figure 7. The nominal cover on the longitudinal reinforcement is 30 mm.

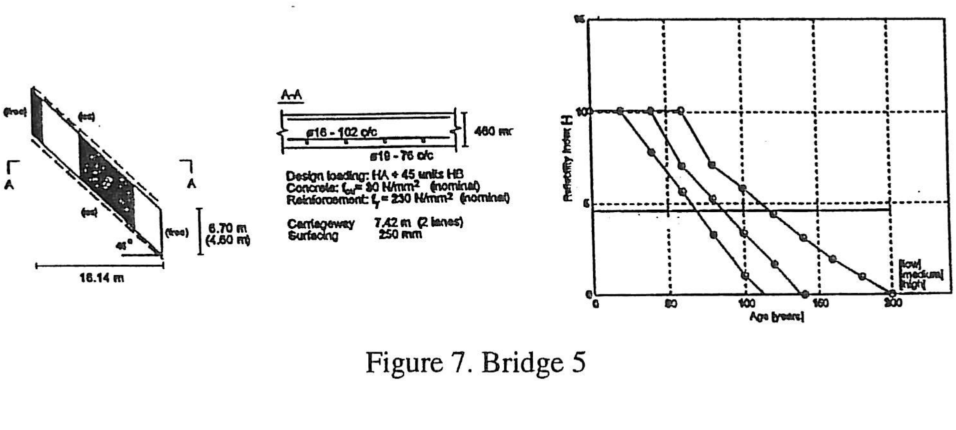

Figure 7. Bridge 5

Bridge 6

Bridge 6 carries the A14 trunk road over a minor road. The bridge was constructed in 1975, the span is 16.2 m, the width 26.1 with a skew of 12 degrees. The bridge is designed to carry 45HB loading. The main dimensions and material data used in the analysis model are shown in Figure 8. The nominal cover on the longitudinal reinforcement is 40 mm.

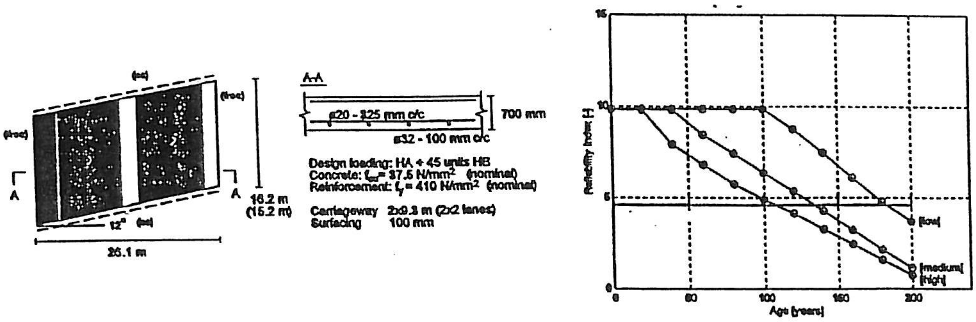

Figure 8. Bridge 6

Acknowledgement

The author would like to thank the Highways Agency, London and CSRconsult, Aalborg, Denmark for permission to publish this paper. The author is particularly grateful to Dr. P. Das of the Highways Agency for his contributions to the fundamental philosophy of this work. However, any views expressed in this paper are not necessarily those of the Highways Agency of the U.K. Department of Transport.

References

CORROSION, Version 1.0 , manual and software. CSR-software, September 1995.

Madsen, H.O. :"*Omission Sensitivity Factors*", Structural Safety, Vol.5, 1988, pp.33-45.

Middleton, C.: "*Example Collapse & Reliability Analyses of Concrete Bridges using a new Analyses Technique*". Cambridge University, 1994.

RELIAB01, Version 2.0 , manual and software. CSR-software, March 1994.

RELIAB02, Version 2.0 , manual and software. CSR-software, April 1994.

Thoft-Christensen, P.: "*Reliability of Plastic Slabs*", ICOSSAR Conference, San Francisco, USA, August 1989.

Thoft-Christensen, P. , F. M. Jensen, C.Middleton & A. Blackmore: *Assessment of the Reliability of Concrete Slab Bridges"*. IFIP WG7.5 Conference, Boulder, USA, April 2-4, 1996.

Thoft-Christensen, P: "*Re-Assessment of Concrete Bridges*". ASCE Structures Congress, Chicago, USA, April 14-18, 1996.

Thoft-Christensen, P. & C. Middleton: "*Reliability Asessment of Concrete Bridges*". Proc. Symposium on "The Safety of Bridges", London, July 4-5, 1996.

Thoft-Christensen, P. & F.M. Jensen: *"Revision of the Bridge Assessment Rules Based on Whole Life Performance: Concrete Bridges"*, Document prepared for the Highways Agency, London by CSRconsult, Aalborg, Denmark, December 1996.

Thoft-Christensen, P.: "*Estimation of the Service Lifetime of Concrete Bridges*", ASCE Structures Congress, Portland, Oregon, April 1997, pp. 248-252.

STRUCTURAL RELIABILITY THEORY SERIES

PAPER NO. 136: H. U. Köylüoğlu, S. R. K. Nielsen & A. Ş. Çakmak: *Solution Methods for Structures with Random Properties subject to Random Excitation.* ISSN 0902-7513 R9444.

PAPER NO. 137: J. D. Sørensen, M. H. Faber & I. B. Kroon: *Optimal Reliability-Based Planning of Experiments for POD Curves.* ISSN 0902-7513 R9455.

PAPER NO. 138: S.R.K. Nielsen & P.S. Skjærbæk, H.U. Köylüoğlu & A.Ş. Çakmak: *Prediction of Global Damage and Reliability based upon Sequential Identification and Updating of RC Structures subject to Earthquakes.* ISSN 0902-7513 R9505.

PAPER NO. 139: R. Iwankiewicz, S. R. K. Nielsen & P. S. Skjærbæk: *Sensitivity of Reliability Estimates in Partially Damaged RC Structures subject to Earthquakes, using Reduced Hysteretic Models.* ISSN 0902-7513 R9507.

PAPER NO. 140: R. C. Micaletti, A. Ş. Çakmak, S. R. K. Nielsen & H. U. Köylüoğlu: *Error Analysis of Statistical Linearization with Gaussian Closure for Large Degree-of-Freedom Systems.* ISSN 1395-7953 R9631.

PAPER NO 141: H. U. Köylüoğlu, S. R. K. Nielsen & A. Ş. Çakmak: *Uncertain Buckling Load and Reliability of Columns with Uncertain Properties.* ISSN 0902-7513 R9524.

PAPER NO. 142: S. R. K. Nielsen & R. Iwankiewicz: *Response of Non-Linear Systems to Renewal Impulses by Path Integration.* ISSN 0902-7513 R9512.

PAPER NO. 143: H. U. Köylüoğlu, A. Ş. Çakmak & S. R. K. Nielsen: *Midbroken Reinforced Concrete Shear Frames Due to Earthquakes. - A Hysteretic Model to Quantify Damage at the Storey Level.* ISSN 1395-7953 R9630.

PAPER NO. 144: S. Engelund: *Probabilistic Models and Computational Methods for Chloride Ingress in Concrete.* Ph.D.-Thesis. ISSN 1395-7953 R9707.

PAPER NO. 145: H. U. Köylüoğlu, S. R. K. Nielsen, Jamison Abbott & A. Ş. Çakmak: *Local and Modal Damage Indicators for Reinforced Concrete Shear Frames subject to Earthquakes.* ISSN 0902-7513 R9521

PAPER NO. 146: P. H. Kirkegaard, S. R. K. Nielsen, R. C. Micaletti & A. Ş. Çakmak: *Identification of a Maximum Softening Damage Indicator of RC-Structures using Time-Frequency Techniques.* ISSN 0902-7513 R9522.

PAPER NO. 147: R. C. Micaletti, A. Ş. Çakmak, S. R. K. Nielsen & P. H. Kirkegaard: *Construction of Time-Dependent Spectra using Wavelet Analysis for Determination of Global Damage.* ISSN 0902-7513 R9517.

PAPER NO. 148: H. U. Köylüoğlu, S. R. K. Nielsen & A. Ş. Çakmak: *Hysteretic MDOF Model to Quantify Damage for TC Shear Frames subject to Earthquakes.* ISSN 1395-7953 R9601.

PAPER NO. 149: P. S. Skjærbæk, S. R. K. Nielsen & A. Ş. Çakmak: *Damage Location of Severely Damaged RC-Structures based on Measured Eigenperiods from a Single Response.* ISSN 0902-7513 R9518.

STRUCTURAL RELIABILITY THEORY SERIES

PAPER NO. 150: S. R. K. Nielsen & H. U. Köylüoğlu: *Path Integration applied to Structural Systems with Uncertain Properties.* ISSN 1395-7953 R9602.

PAPER NO. 151: H. U. Köylüoğlu & S. R. K. Nielsen: *System Dynamics and Modified Cumulant Neglect Closure Schemes.* ISSN 1395-7953 R9603.

PAPER NO. 152: R. C. Micaletti, A. Ş. Çakmak, S. R. K. Nielsen, H. U. Köylüoğlu: *Approximate Analytical Solution for the 2nd-Order moments of a SDOF Hysteretic Oscillator with Low Yield Levels Excited by Stationary Gaussian White Noise.* ISSN 1395-7953 R9715.

PAPER NO. 153: R. C. Micaletti, A. Ş. Çakmak, S. R. K. Nielsen & H. U. Köylüoğlu: *A Solution Method for Linear and Geometrically Nonlinear MDOF Systems with Random Properties subject to Random Excitation.* ISSN 1395-7953 R9632.

PAPER NO. 154: J. D. Sørensen, M. H. Faber, I. B. Kroon: *Optimal Reliability-Based Planning of Experiments for POD Curves.* ISSN 1395-7953 R9542.

PAPER NO. 155: J. D. Sørensen, S. Engelund: *Stochastic Finite Elements in Reliability-Based Structural Optimization.* ISSN 1395-7953 R9543.

PAPER NO. 156: C. Pedersen, P. Thoft-Christensen: *Guidelines for Interactive Reliability-Based Structural Optimization using Quasi-Newton Algorithms.* ISSN 1395-7953 R9615.

PAPER NO. 157: P. Thoft-Christensen, F. M. Jensen, C. R. Middleton, A. Blackmore: *Assessment of the Reliability of Concrete Slab Bridges.* ISSN 1395-7953 R9616.

PAPER NO. 158: P. Thoft-Christensen: *Re-Assessment of Concrete Bridges.* ISSN 1395-7953 R9605.

PAPER NO. 159: H. I. Hansen, P. Thoft-Christensen: *Wind Tunnel Testing of Active Control System for Bridges.* ISSN 1395-7953 R9662.

PAPER NO 160: C. Pedersen: *Interactive Reliability-Based Optimization of Structural Systems.* Ph.D.-Thesis. ISSN 1395-7953 R9638.

PAPER NO. 161: S. Engelund, J. D. Sørensen: *Stochastic Models for Chloride-initiated Corrosion in Reinforced Concrete.*ISSN 1395-7953 R9608.

PAPER NO. 165: P. H. Kirkegaard, F. M. Jensen, P. Thoft-Christensen: *Modelling of Surface Ships using Artificial Neural Networks.* ISSN 1593-7953 R9625.

PAPER NO. 166: S. R. K. Nielsen, S. Krenk: *Stochastic Response of Energy Balanced Model for Wortex-Induced Vibration.* ISSN 1395-7953 R9710.

PAPER NO. 167: S.R.K. Nielsen, R. Iwankiewicz: *Dynamic systems Driven by Non-Poissonian Impulses: Markov Vector Approach.* ISSN 1395-7953 R9705.

PAPER NO. 168: P. Thoft-Christensen: *Lifetime Reliability Assessment of Concrete Slab Bridges.* ISSN 1395-7953 R9717.

Department of Building Technology and Structural Engineering
Aalborg University, Sohngaardsholmsvej 57, DK 9000 Aalborg
Telephone: +45 9635 8080 Telefax: +45 9814 8243